Final Report

San Francisquito Creek
Hydraulic Modeling and Floodplain Mapping

Volume I: Channel Hydraulic Modeling

Prepared For:

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San Francisco District

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January 22, 2010

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1 INTRODUCTION

The San Francisquito Creek watershed encompasses an area of approximately 45 square miles, extending from the ridge of the Santa Cruz Mountains to the San Francisco Bay in California. Located on the border of Santa Clara and San Mateo Counties, San Francisquito Creek begins at the confluence of Corte Madera Creek and Bear Creek at Jasper Ridge Preserve of Stanford University and flows into San Francisco Bay approximately 2.5 miles south of the Dumbarton Bridge. Flooding on the creek affects the cities of Menlo Park and East Palo Alto in San Mateo County, and Palo Alto in Santa Clara County.

A steady HEC-RAS model was developed by Noble Consultants, Inc. (2009) for San Francisquito Creek. The modeled reach was from the mouth of the creek to approximately one mile upstream of Highway 280, with a total length of approximately 55,000 feet. This steady hydraulic model was calibrated and verified using three historic flood events, and was used to estimate the flow capacities of the creek at different reaches and of the bridges on the creek.

The purpose of this present study is to produce and/or update the existing floodplain mapping along the San Francisquito Creek from HWY 280 to the San Francisco Bay. The previous steady HEC-RAS model (NCI, 2009) was converted to an unsteady model in this study by applying the flow hydrographs and representing the potential flow breakout locations as lateral structures. A coincident frequency analysis (CFA) was performed by the Corps based on this unsteady model. The statistical water stages coincident with the return flow discharges were determined in the CFA for the index location which is located at Station 2400. This index location was used as the downstream limit of the modeled reach in the unsteady HEC-RAS model for estimating breakout flow hydrographs along San Francisquito Creek.

The breakout flow hydrographs along San Francisquito Creek were computed for eight flood events with return periods of 2, 5, 10, 25, 50, 100, 250, and 500 years, respectively. These breakout flow hydrographs were input into the FLO-2D model as inflow hydrographs. The floodplain modeling was then conducted using the FLO-2D model, based on which, the floodplain maps were generated in ArcMap.

The coincident frequency analysis was documented by the Corps (USACE, 2010). The floodplain modeling and mapping were presented in the report prepared by Northwest Hydraulic Consultants, Inc (NHC, 2010). This report only summarizes the development of the unsteady HEC-RAS model, and the computation of the breakout flow hydrographs with the unsteady model.

It is noted that the horizontal coordinate system used in the HEC-RAS model and in this study is the California State Plane NAD83, Zone 3, in US Survey Feet. The vertical datum is NAVD88, feet. All model inputs and results are referenced to this datum.

2 DEVELOPMENT AND CALIBRATION OF THE UNSTEADY HEC-RAS MODEL

The steady HEC-RAS model (NCI, 2009) was converted into a unsteady model with the following two major revisions: (1) replacing the peak flow discharges at the upstream boundary and at the flow change locations with flow hydrographs, and (2) adding lateral structures (weirs) along the banks of the creek for the locations where breakout flow may potentially occur.

The return peak flow rates along San Francisquito Creek were determined in the SCVWD (2007) hydrology report. The peak flow rates and the drainage areas for different locations along the creek were provided to us by the Corps and are listed in Table 2-1. The flow hydrographs based on the associated SCVWD HEC-1 models were also provided by the Corps for use in the unsteady HEC-RAS model. Four flow change locations were identified within the modeled reach in the HEC-RAS model. They include the locations downstream of Los Trancos Creek, at El Camino Real, at US 101 Bridge, and at Palo Alto of Santa Clara County Airport. The flow hydrographs at these locations were used as the upstream flow boundary condition and the lateral inflow hydrographs, respectively, in the unsteady HEC-RAS model. It is noted that only one flow change location (the confluence of Los Trancos Creek) was included in the previous steady HEC-RAS model.

Table 2-1. Return Peak Flow Rates along Guadalupe River

San Francisquito Creek and Tributaries Location	Drainage Area (mi^2)	Peak Flow Rate (cfs)							
		2-year[2]	5-year[1]	10-year[1]	25-year[1]	50-year[1]	100-year[1]	250-year[2]	500-year[2]
Bear Creek u/s San Francisquito Creek (S8)	11.85	-	1,100	1,600	2,200	2,700	3,200	-	-
San Francisquito Creek u/s Lake Searsville S8A)	14.65	-	1,500	2,100	2,900	3,500	4,100	-	-
San Francisquito Creek d/s Lake Searsville	14.65	-	1,500	2,100	2,900	3,500	4,100	-	-
San Francisquito Creek d/s Bear Creek (S8B)	26.50	1599	2,700	3,800	5,200	6,300	7,300	8,705	9,836
San Francisquito Creek u/s Los Trancos Creek (S9)	29.61	1599	2,800	3,900	5,400	6,500	7,600	9,161	10,429
Los Trancos Creek u/s San Francisquito Creek (G6)	7.65	-	380	560	810	1,000	1,200	-	-
San Francisquito Creek d/s Los Trancos Creek (S9A)	37.26	1799	3,200	4,500	6,200	7,500	8,800	10,652	12,162
San Francisquito Creek @ USGS 11164500 (S10)	37.62	1799	3,200	4,500	6,200	7,500	8,800	10,652	12,162
San Francisquito Creek @ El Camino Real (S12)	41.20	1899	3,300	4,700	6,500	7,900	9,200	11,037	12,523
San Francisquito Creek @ US 101 (S14)	44.55	1899	3,400	4,800	6,600	8,000	9,300	11,133	12,614
San Francisquito Creek @ Palo Alto of Santa Clara County Airport (S14)	46.17	2099	3,600	5,000	6,800	8,100	9,400	11,230	12,705

Source: This table was provided by the Corps.

[1]: Reported Values from SCVWD (2007) San Francisquito Creek Hydrology Report, Revised December 2007.

[2]: USACE (CESPN-ET-EW) Calculated Using FDA - Exceedance Probabiliity Funcitions with Uncertainty Graphical Method (N=25yr).

The cross sections included in the steady HEC-RAS model mainly covered the main channel of San Francisquito Creek. The banks of these cross sections are either represented by the tops of grounds for the incised channels or by levees and floodwalls for the reaches protected by levees and floodwalls. When the water stage is higher than the bank elevation, part of the flood water will flow into the floodplain areas along the creek. In some cases, water will overtop the banks of the channel and permanently leave the creek (referred as breakout flow in this analysis) during the flood events. This is particularly true for the channels protected by levees and floodwalls. The breakout flow was not included in the previous steady HEC-RAS model (NCI, 2009).

The breakout flow was modeled in this unsteady HEC-RAS model by adding lateral structures (weirs) along the banks of the creek for all the locations where the water levels may potentially exceed the bank elevations and thus breakout flow may occur. The weir stations and crest elevations for each lateral weir were determined based on the cross section data included in the steady HEC-RAS model. The tail water connection type of the lateral weirs was set to “out of the system” assuming the breakout flow will permanently leave the creek. The weir coefficient was set to 2.6 for the locations where levees or floodwalls exist. This is the lower end of the typical overflow weir coefficient range (2.6 to 3.1) recommended in the HEC-RAS Hydraulic Reference Manual (USACE, 2008). When the overflow occurs at an incised channel, the vegetation, obstruction and/or the building on the top grounds where the overflow will pass through present larger resistance to the flow compared to the typical overflow weirs. A lower weir coefficient of 2.0 was thus assigned for the possible flow breakout locations where the incise channel exists.

The unsteady HEC-RAS model was re-calibrated using the same historic flood events that were used in the calibration of the steady model. These three flood events occurred on February 13, 2000, December 16, 2002, and January 1, 2006, respectively. The Manning’s roughness values used in the steady model were reduced by 5% in the unsteady model in order to obtain a reasonable agreement between the model results predicted by the unsteady model and the high water mark data measured for these flood events. The calibrated Manning’s roughness values for the main channel range between 0.0285 for the lower reach and 0.0408 for the middle and upper reaches. The floodplain roughness coefficients are generally larger than the main channel by 0.019. These calibrated Manning's roughness coefficients are consistent with values recommended in the HEC-RAS Hydraulic Reference Manual (USACE, 2008). The recommended Manning's roughness values vary between 0.025 and 0.04 for a natural (tidal) stream with some vegetation and with the creek bottom consisting of bay mud, sand and cobbles, and between 0.03 and 0.05 for mountain creeks with gravels, cobbles, a few boulders on the channel bottom, and steep banks with trees and brush on submerged bank.

The comparisons of the water levels between the model results and the measured data are summarized in Table 2-2 for the three flood events used in the calibration. The comparisons are also shown in Figure 2-1 through Figure 2-4 for the four locations where the high water marks were recorded. Both the model results predicted by the unsteady model and the results predicted by the steady model are shown in these figures. The water levels predicted by the unsteady HEC-RAS model are similar to the results of the steady model, both showing good agreement with the measured data for all the three flood events at three of the four locations: Hwy 101 Bridge, Waverley Bike Bridge, and the USGS Gage Station. The discrepancy between the model results and the high water mark data at the Pope/Chaucer Bridge might be caused by the questionable accuracy of the measured high water mark data, as discussed in the previous study (NCI, 2009).

Table 2-2. Comparison of Water Stages between Data and Unsteady Model

Location	Representative Sta. in HEC-RAS Model	Water Levels (ft, NAVD88)					
		2/13/2000 (4,010 cfs)		12/16/2002 (3,730 cfs)		1/1/2006 (4,840 cfs)	
		Data	Model	Data	Model	Data	Model
Upstream of Hwy 101	Sta. 80+27	+16.9	+16.8	+16.2	+16.4	+18.2	+18.0
Pope/Chaucer Bridge	Sta. 178+37	+38.1	+40.4	+39.7	+39.4	+43.4	+42.9
Waverley Bike Bridge	Sta. 249+00	+55.9	+55.7	+55.3	+55.0	+57.4	+57.8
USGS Gage Station	Sta. 405+61	+121.8	+122.0	-	+121.6	-	123.1

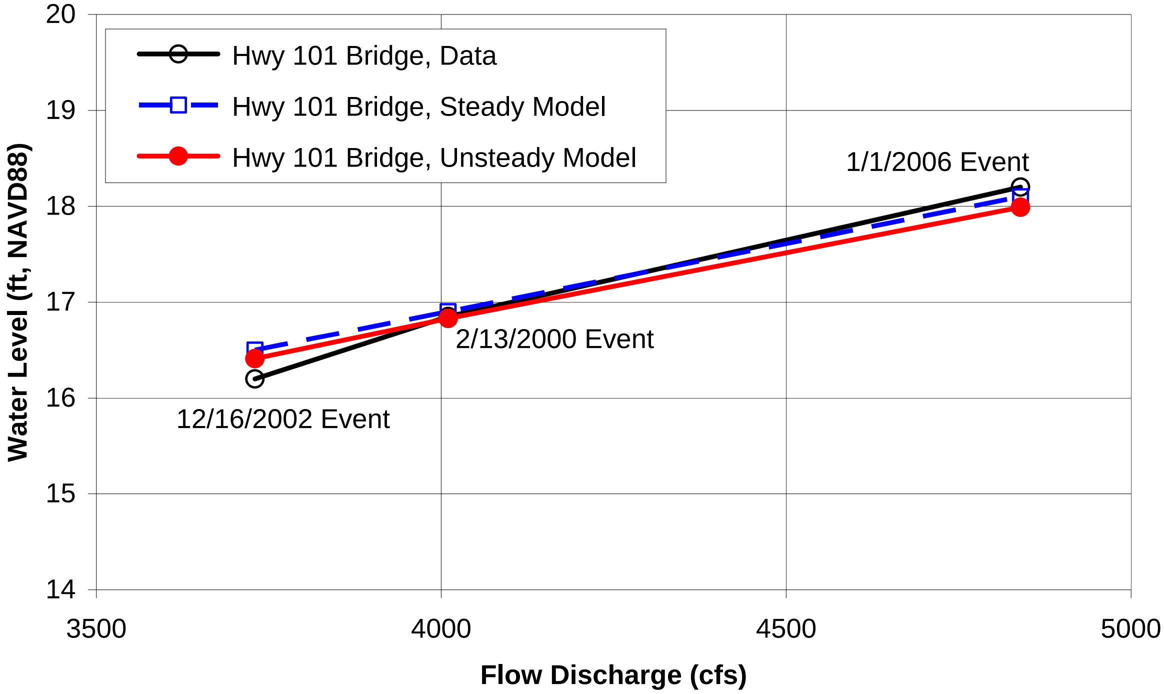


Figure 2-1. Predicted Water Levels versus Data at the Hwy 101 Bridge

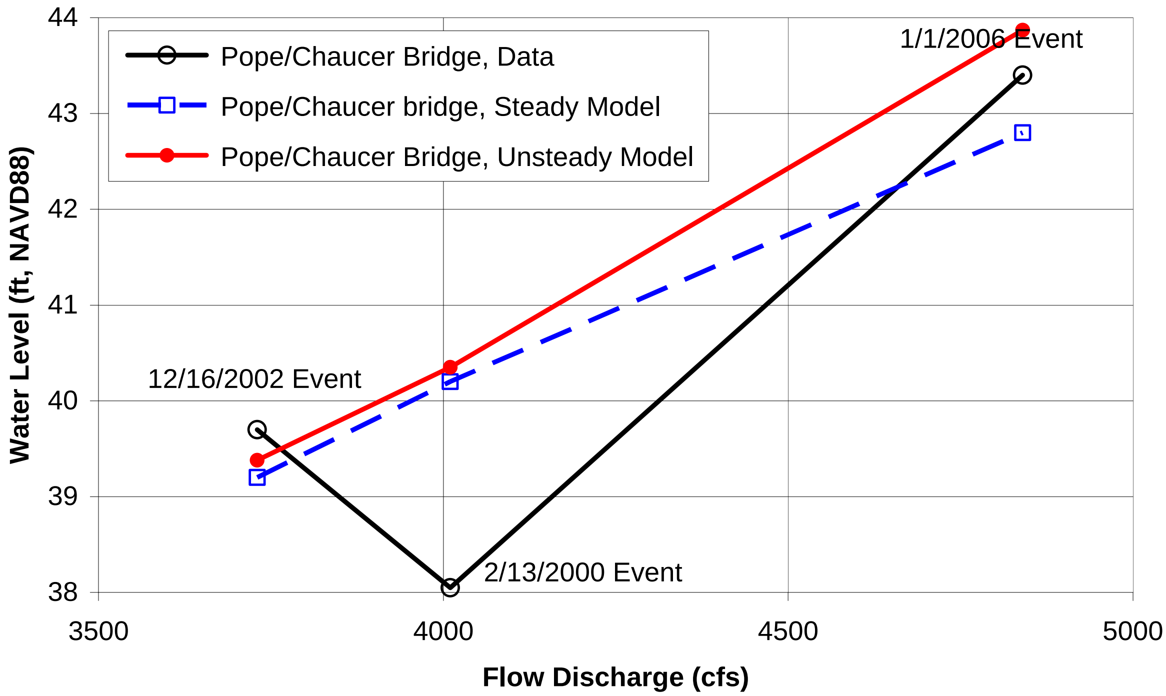


Figure 2-2. Predicted Water Levels versus Data at Pope/Chaucer Bridge

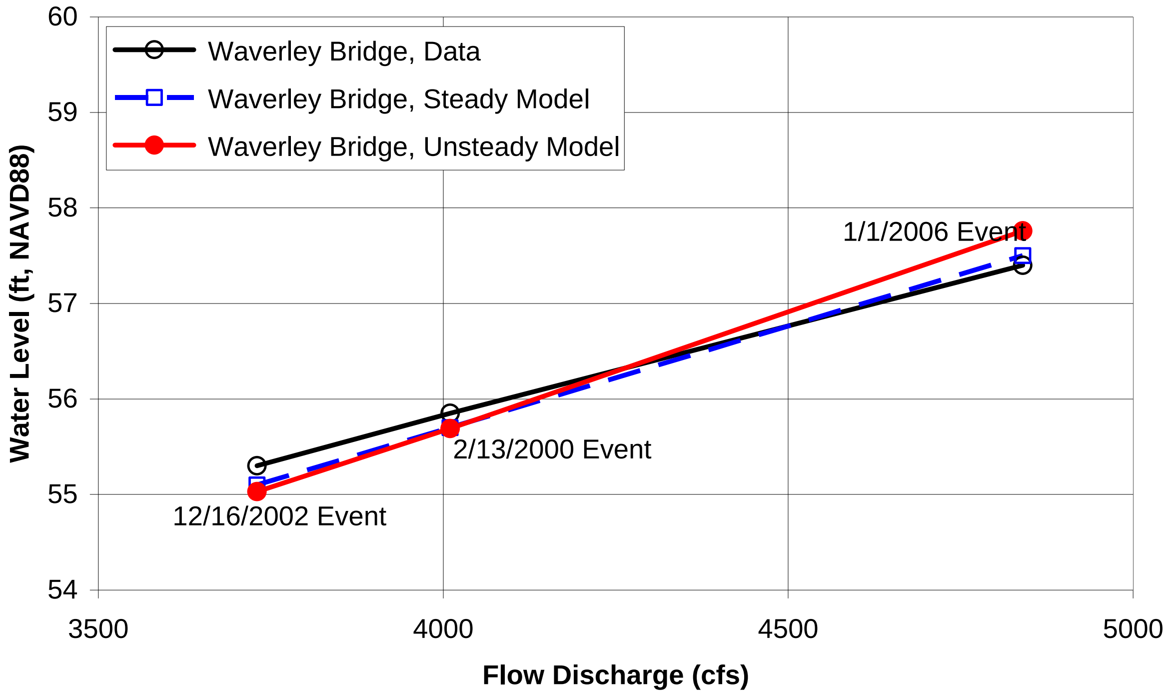


Figure 2-3. Predicted Water Levels versus Data at Waverley Bike Bridge

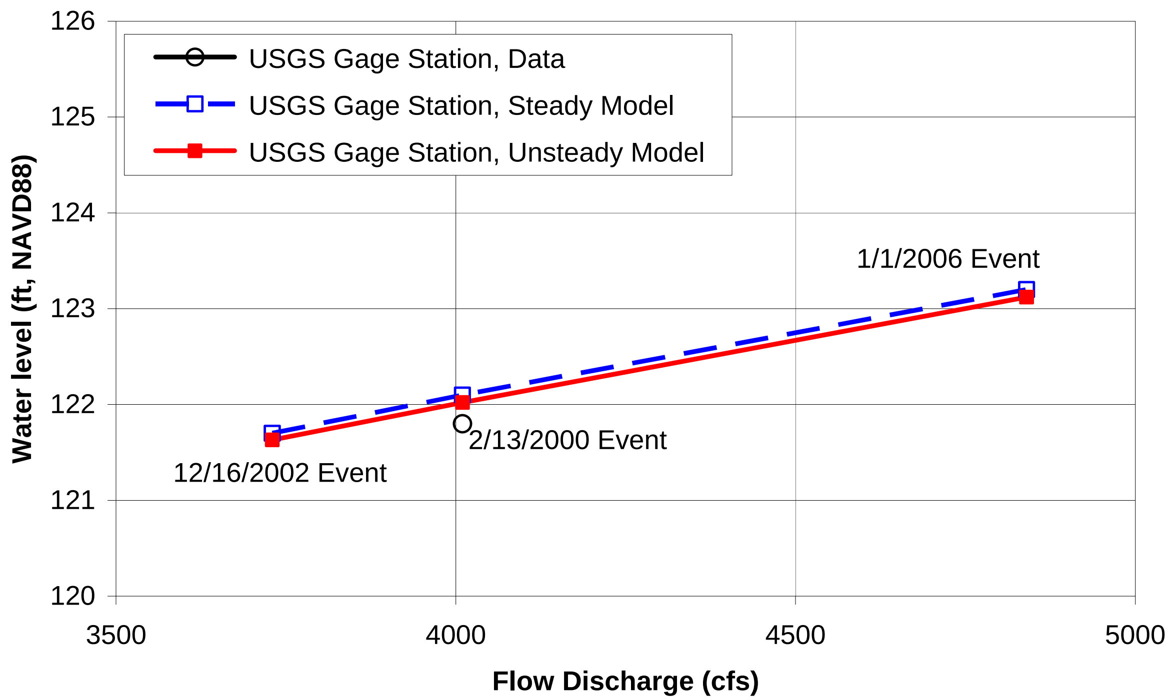


Figure 2-4. Predicted Water Levels versus Data at USGS Gage Station

3 BREAKOUT FLOW HYDROGRAPHS

When the water levels in the creek exceed the top banks or top grounds of the creek, water may breakout from the banks or top grounds, leave the creek, and flow into the floodplain areas. In the floodplain modeling with the FLO-2D model, the breakout flow hydrographs along the creek are required as the inflow hydrographs for the FLO-2D model. Eight flood events with the return intervals of 2, 5, 10, 25, 50, 100, 250, and 500 years, respectively, were simulated with the unsteady HEC-RAS model in order to determine the breakout flow hydrographs.

3.1 Downstream Boundary Condition

In order to determine the downstream limit of the fluvial flooding and the water levels at this downstream limit for different return flood events, a coincident frequency analysis (CFA) was performed by the Corps with the unsteady HEC-RAS model (USACE, 2010). Based on this coincident frequency analysis, the downstream limit (index station) of the fluvial flooding was determined to be at approximately river station 24+00. The location of this index location is shown in Figure 3-1. The downstream limit of the unsteady HEC-RAS model for the breakout flow computation was then set to this index station. The water stages at this index station were also determined in the coincident frequency analysis for the eight flood events, and were used as the downstream boundary condition for the revised unsteady HEC-RAS model. The water stages at the index station (Sta 24+00) determined in the CFA (USACE, 2010) are listed in Table 3-1.

Table 3-1. Water Stages at Index Station 24+00

Total Probability	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Return Period (Years)	2	5	10	25	50	100	250	500
Water Stage (ft, NAVD88)	8.63	10.08	11.44	11.96	12.05	12.17	12.25	12.31

Figure 3-1. The Location of CFA Index Station 24+00

3.2 Flow Hydrographs along the Creek

The hydrographs for the eight flood events at the upstream boundary of the modeled reach and at the four flow change locations along the creek were provided to us by the Corps (USACE, 2009a). The flow change location at Palo Alto of Santa Clara County Airport is beyond the modeled reach of the HEC-RAS model for the breakout flow hydrograph computation. Therefore, flow hydrographs were specified in the revised unsteady model at the upstream boundary and at the other three flow change locations in the modeled reach. As examples, Figure 3-2 through Figure 3-5 show the flow hydrographs for the 50-, 100-, 250-, and 500-year flood events, respectively, at these four locations.

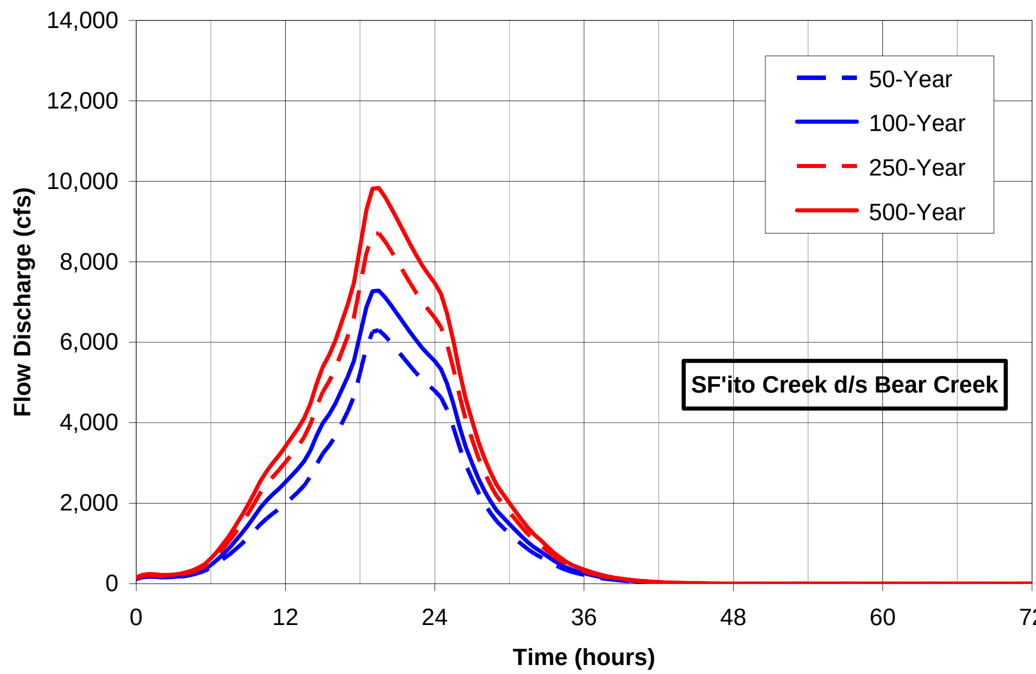


Figure 3-2. Flow Hydrographs of San Francisquito Creek d/s of Bear Creek

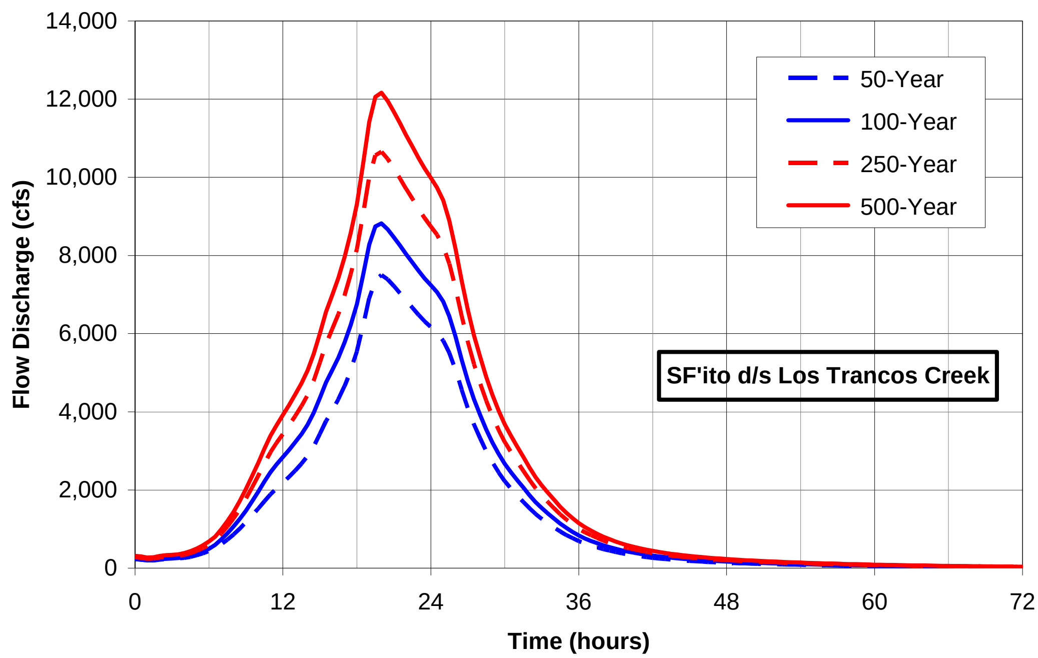


Figure 3-3. Flow Hydrographs of San Francisquito Creek d/s of Los Trancos Creek

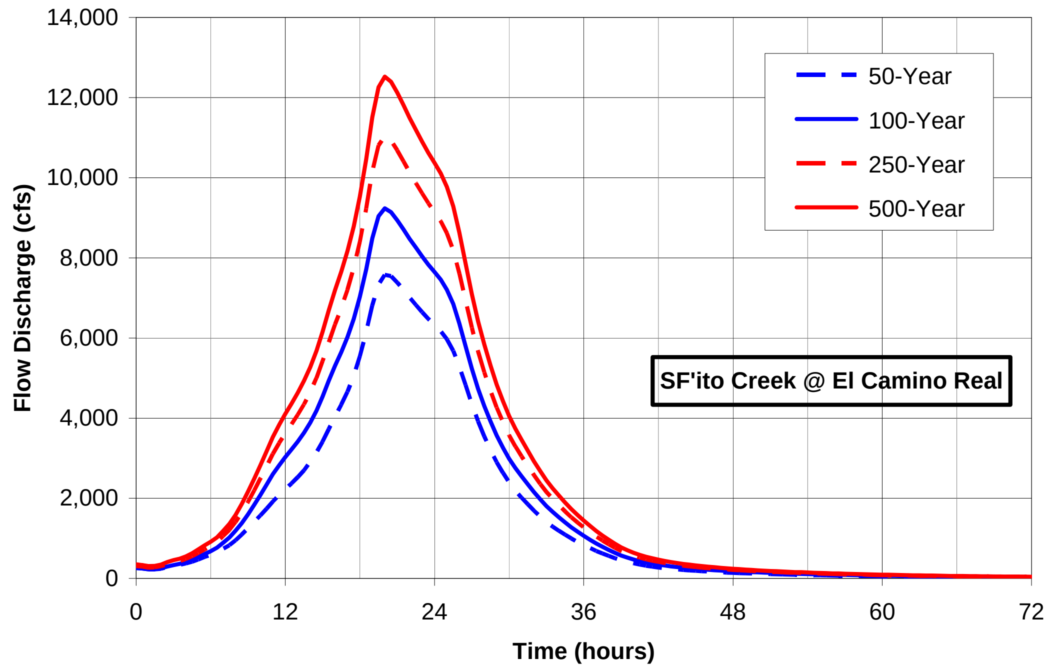


Figure 3-4. Flow Hydrographs of San Francisquito Creek at El Camino Real

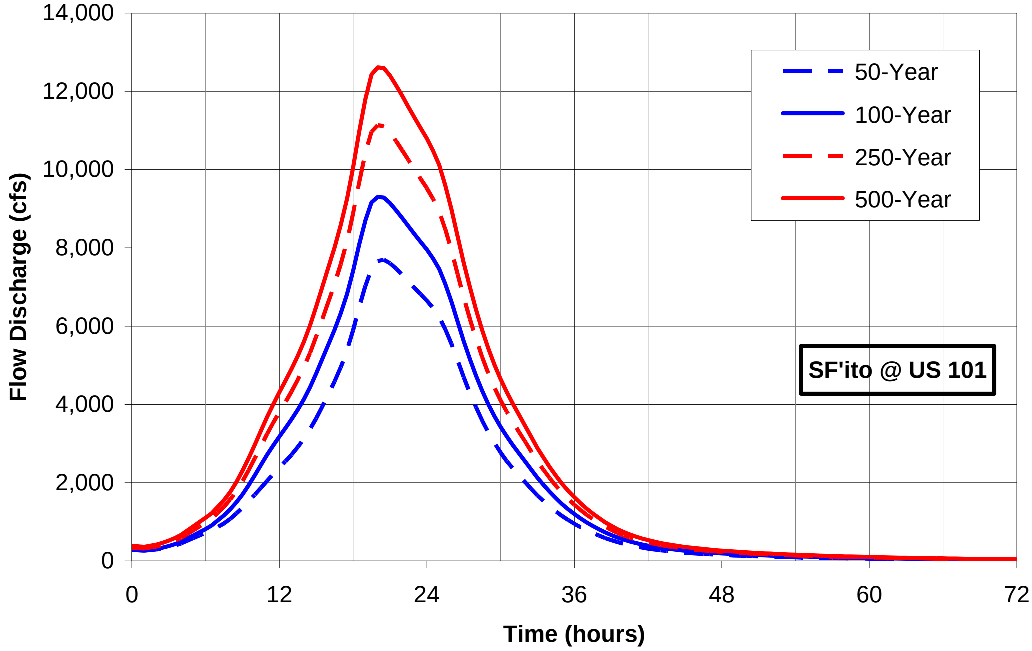


Figure 3-5. Flow Hydrographs of San Francisquito Creek at US 101

3.3 Computed Breakout Flow Hydrographs

Based on the model simulations for the eight flood events, breakout flow is expected to occur at different locations during the flood events with the return intervals of 25 years and longer. Six flow breakout locations were identified for the 500-year flood event, while five locations were identified for the 50-year flood event, and four locations were identified for the 25-year flood event. The six breakout locations for the 500-year flood event are located at the left and right banks upstream of the Middlefield Road Bridge, at the left and right banks upstream of Pope/Chaucer Street Bridge, and at the left and right banks downstream of the Bayshore (US 101) Bridge. Flow breakout will not occur upstream of the Middlefield Road Bridge during the 25-year flood event, and will not occur at the right bank upstream of the Middlefield Road Bridge during the 50-year flood event. The breakout locations for different flood events are shown in the flood inundation maps presented in Volume II of the report (NHC, 2010).

The San Francisquito Creek has limited flow capacity near Middlefield Road and Pope/Chaucer Street. The bridges at these two road crossings present additional flow constriction points along this reach. The openings of these two bridges are very limited compared to the adjacent channel cross sections. As an example, Figure 3-6 shows the upstream face of the Pope/Chaucer Bridge. The limited channel capacity and the bridges with limited openings are the main reasons to cause flow breakout at the banks upstream of the Middlefield Road and Pope/Chaucer Bridges.

Figure 3-6. Limited Opening of the Pope/Chaucer Bridge

The channel downstream of Highway 101 also has inadequate flow capacity. The flow capacity of this lower reach was estimated to be approximately 4,400 cfs (NCI, 2009). The Highway 101 Bridge is an additional constriction point with a flow capacity less than 5,000 cfs. The reach upstream of Highway 101 is protected by floodwalls that are high enough to prevent water overtopping during the flood events. The reach downstream of Highway 101 is protected by earthen levees, as shown in Figure 3-7. However, these levees are not high enough to prevent water overtopping during the extreme flood events. It was estimated in this study that the breakout flow would occur even under the 25-year flood event.

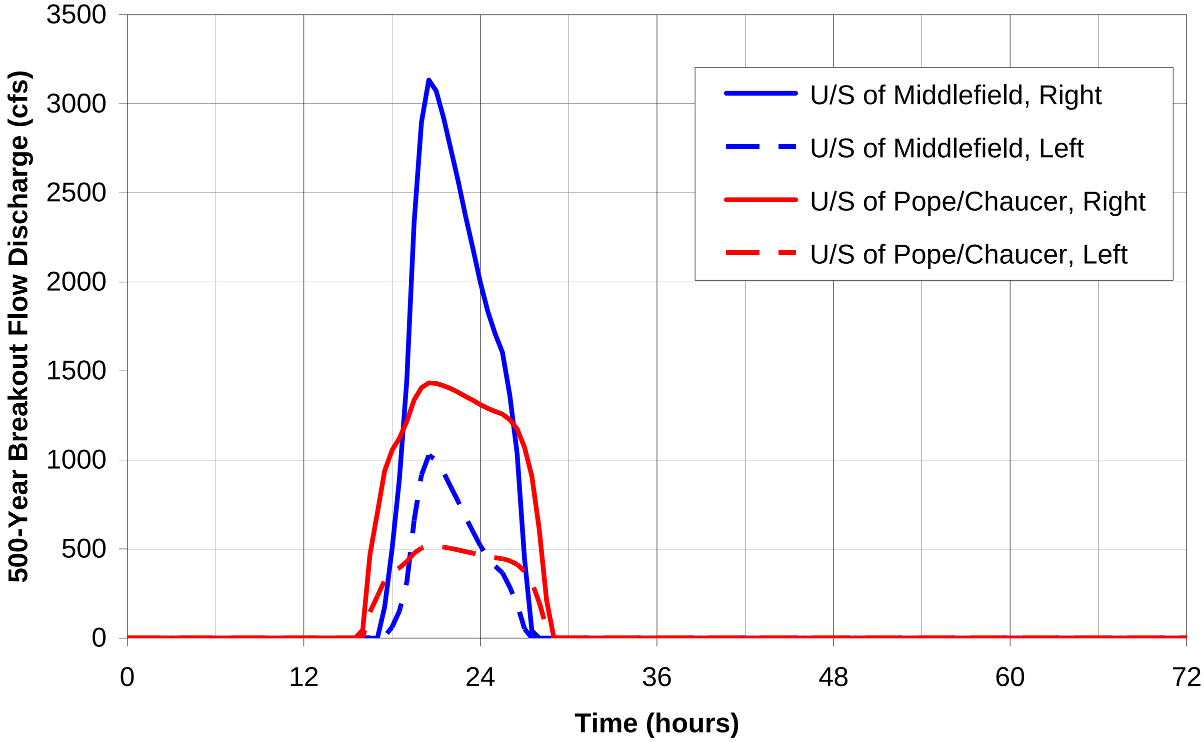
Figure 3-7. Channel downstream of the Highway 101 Crossing (Facing Downstream)

The computed breakout flow hydrographs at different locations are shown in Figure 3-8 through Figure 3-12 for the 500-, 250-, 100-, 50-, and 25-year flood events, respectively. The maximum water surface profiles compared to the bank elevations at these breakout locations are shown in Figure 3-13 through Figure 3-17 for these five flood events, respectively. These breakout flow hydrographs were then used as the inflow hydrographs in the floodplain modeling with the FLO-2D model.

The total water volumes, durations and peak flow rates of the breakout out flow for different flood events are summarized in Table 3-2 for the six breakout locations, respectively.

Table 3-2. Breakout Flow Summary

Breakout Locations	Events	Volume (ac-ft)	Duration (Hours)	Peak Flow Rate (cfs)
u/s Middlefield Road , Right	500-Yr	1,540	11	3,134
	250-Yr	926	9	2,257
	100-Yr	296	7	1,097
	50-Yr	7	2	81
	25-Yr	0	0	0
u/s Middlefield Road, Left	500-Yr	422	10	1,025
	250-Yr	211	9	628
	100-Yr	46	6	206
	50-Yr	0	0	0
	25-Yr	0	0	0
u/s Pope/Chaucer Street, Right	500-Yr	1,173	13	1,433
	250-Yr	959	12	1,339
	100-Yr	661	9	1,177
	50-Yr	285	7	913
	25-Yr	44	3	333
u/s Pope/Chaucer Street, Left	500-Yr	413	13	517
	250-Yr	334	12	477
	100-Yr	227	9	415
	50-Yr	92	7	314
	25-Yr	11	3	93
d/s Hwy 101, Right	500-Yr	328	16	332
	250-Yr	274	14	296
	100-Yr	196	12	259
	50-Yr	126	9	235
	25-Yr	971	7	185
d/s of Hwy 101, Left	500-Yr	835	17	917
	250-Yr	630	15	855
	100-Yr	420	13	786
	50-Yr	240	10	740
	25-Yr	971	8	618



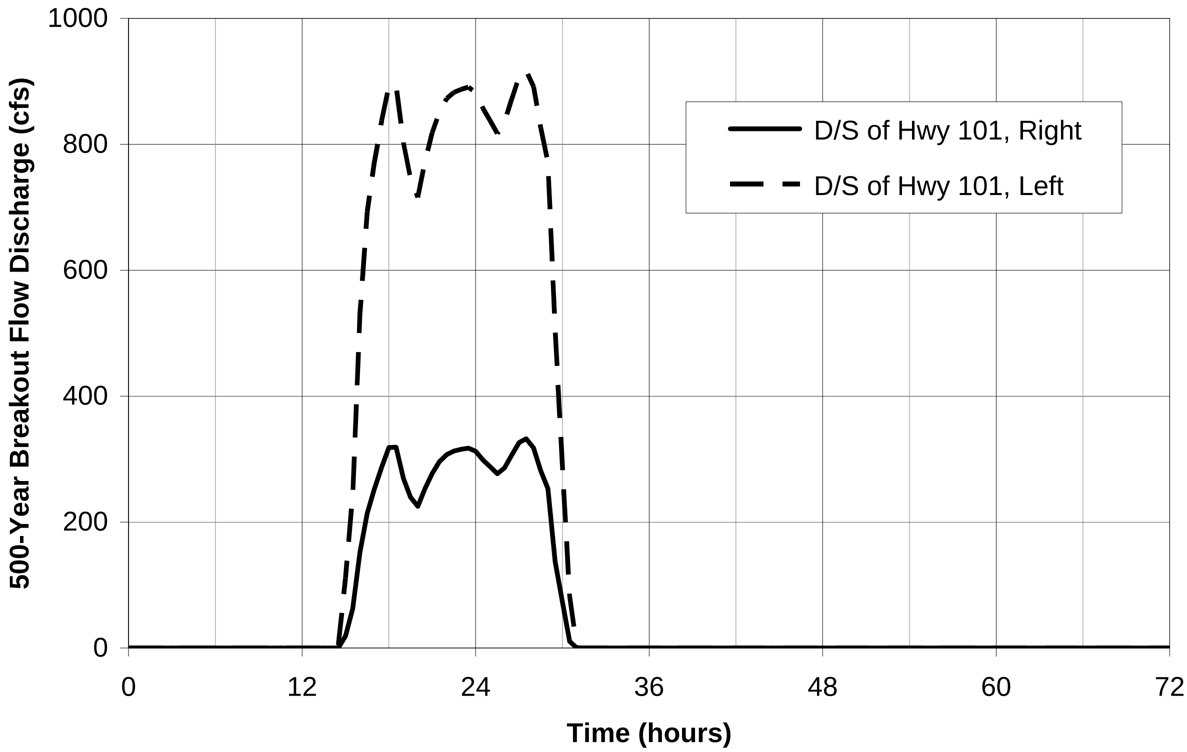
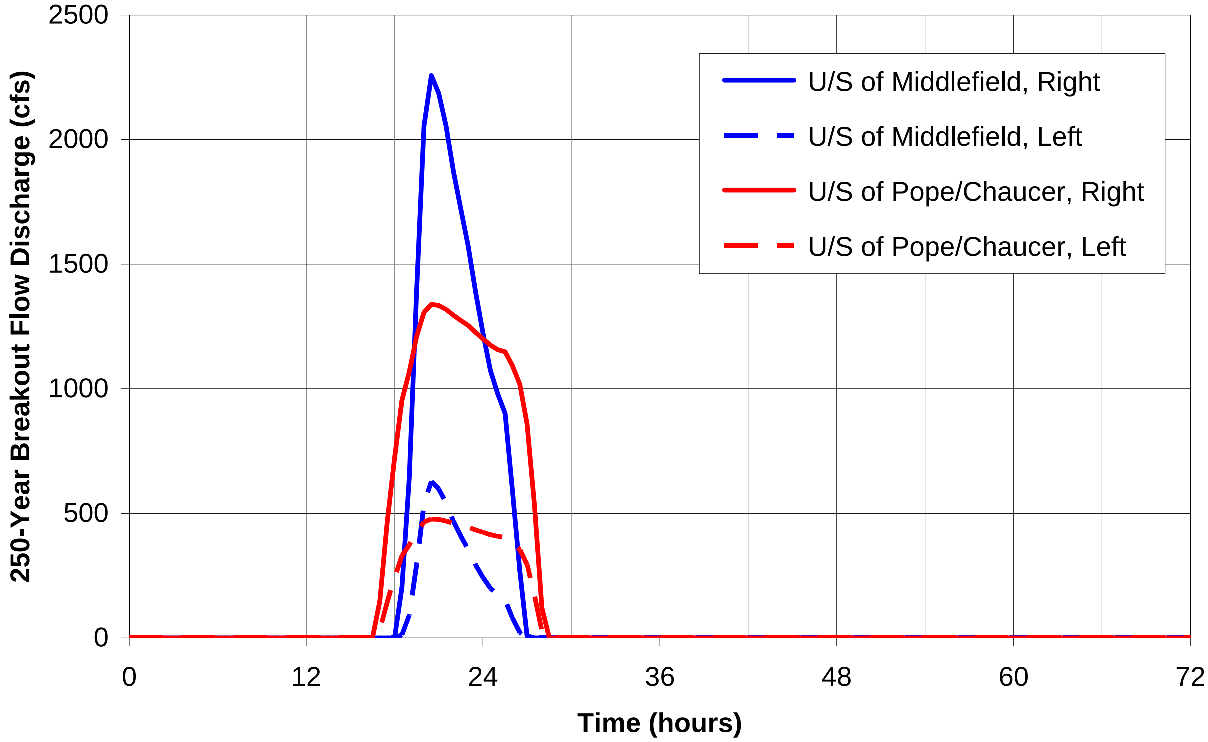


Figure 3-8. Breakout Flow Hydrographs for the 500-Year Flood Event



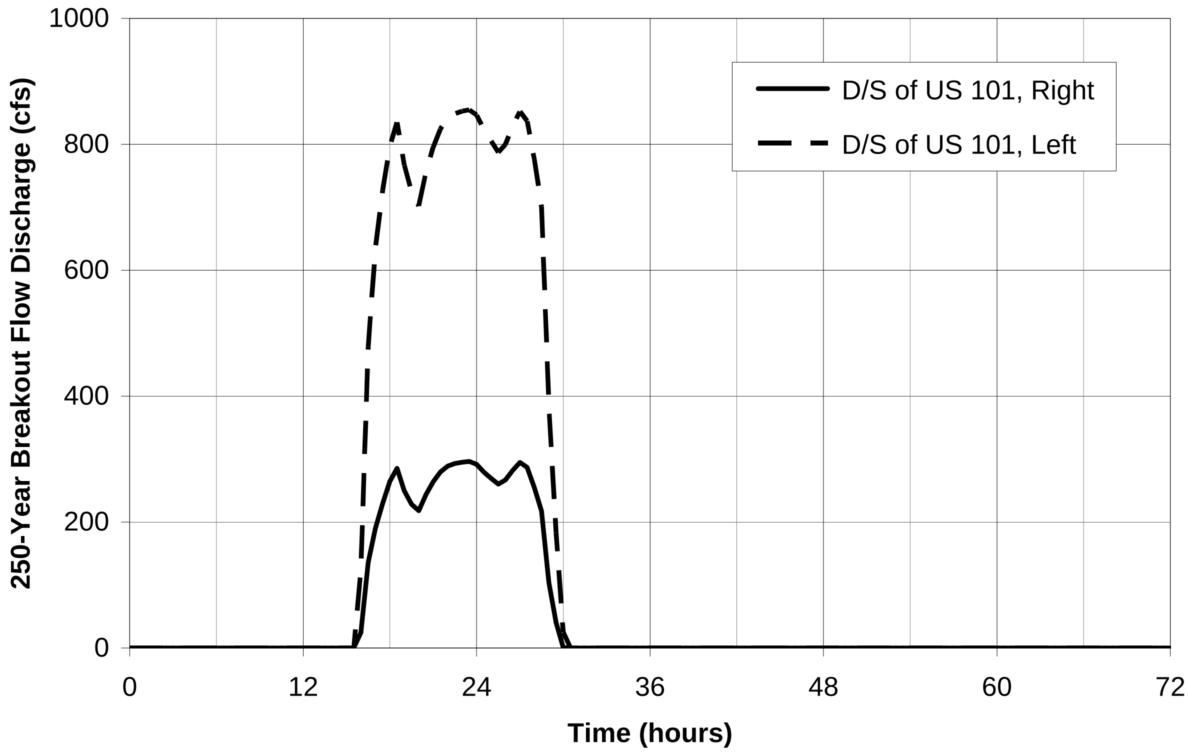
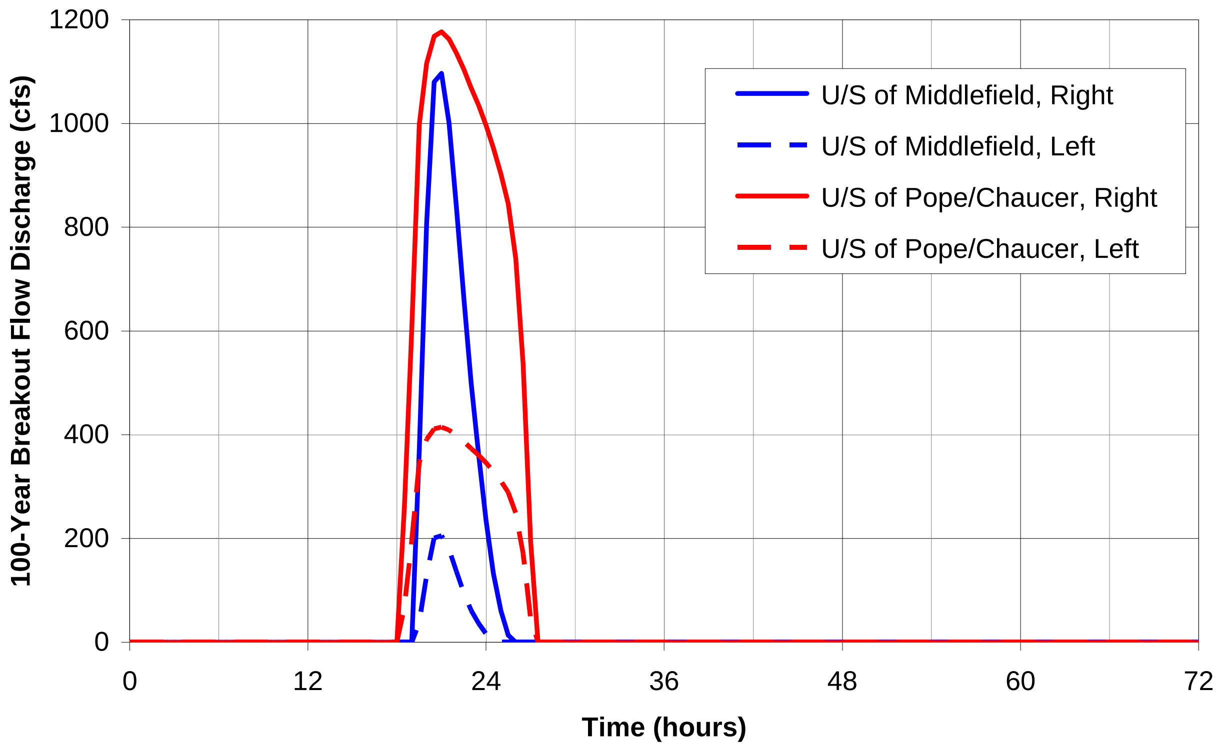


Figure 3-9. Breakout Flow Hydrographs for the 250-Year Flood Event



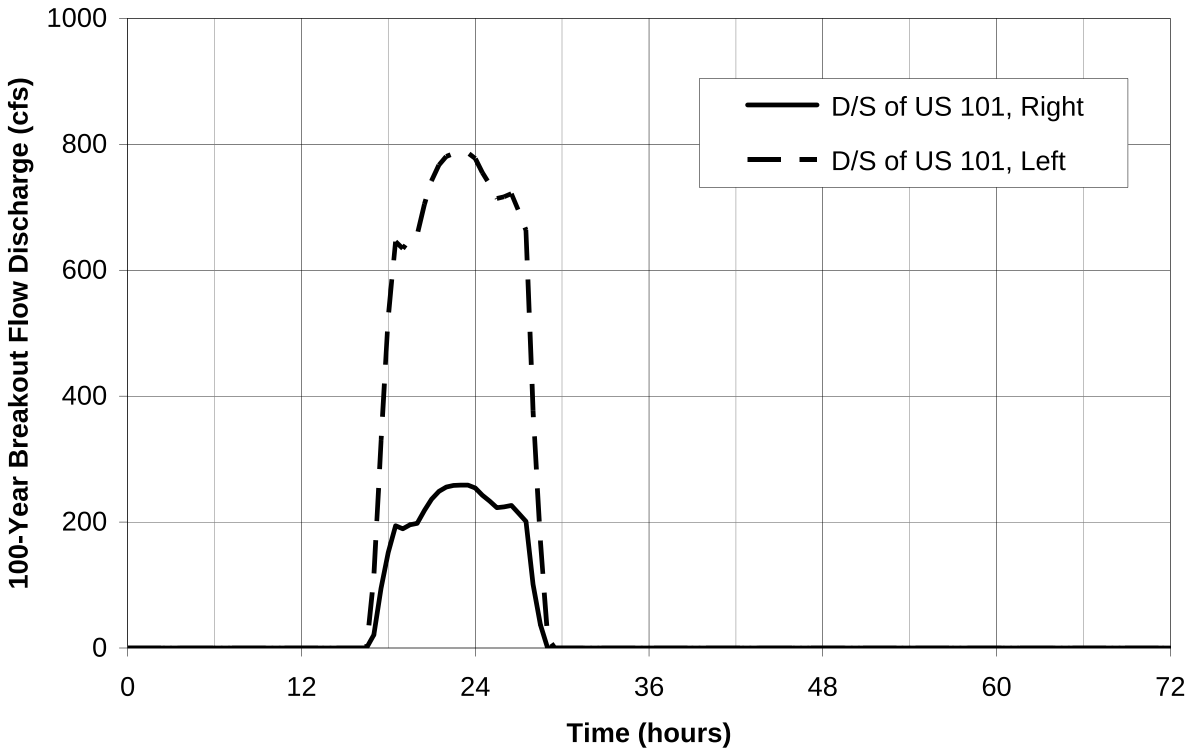
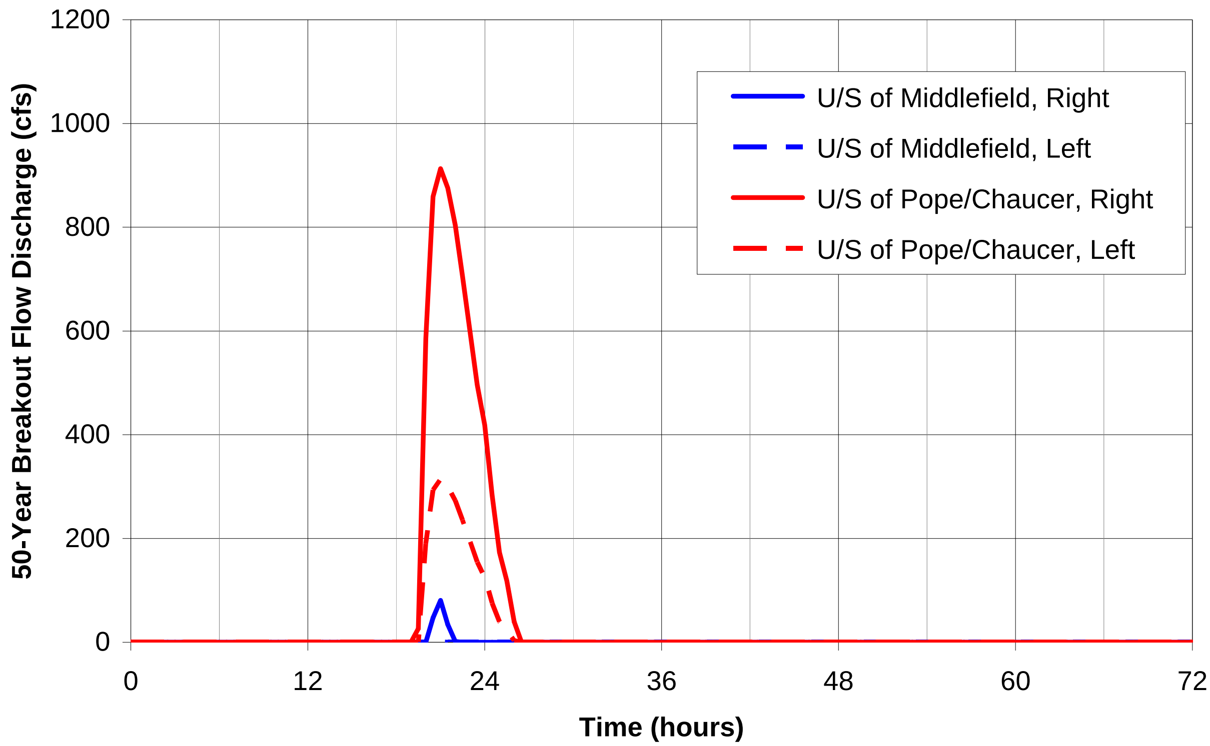


Figure 3-10. Breakout Flow Hydrographs for the 100-Year Flood Event



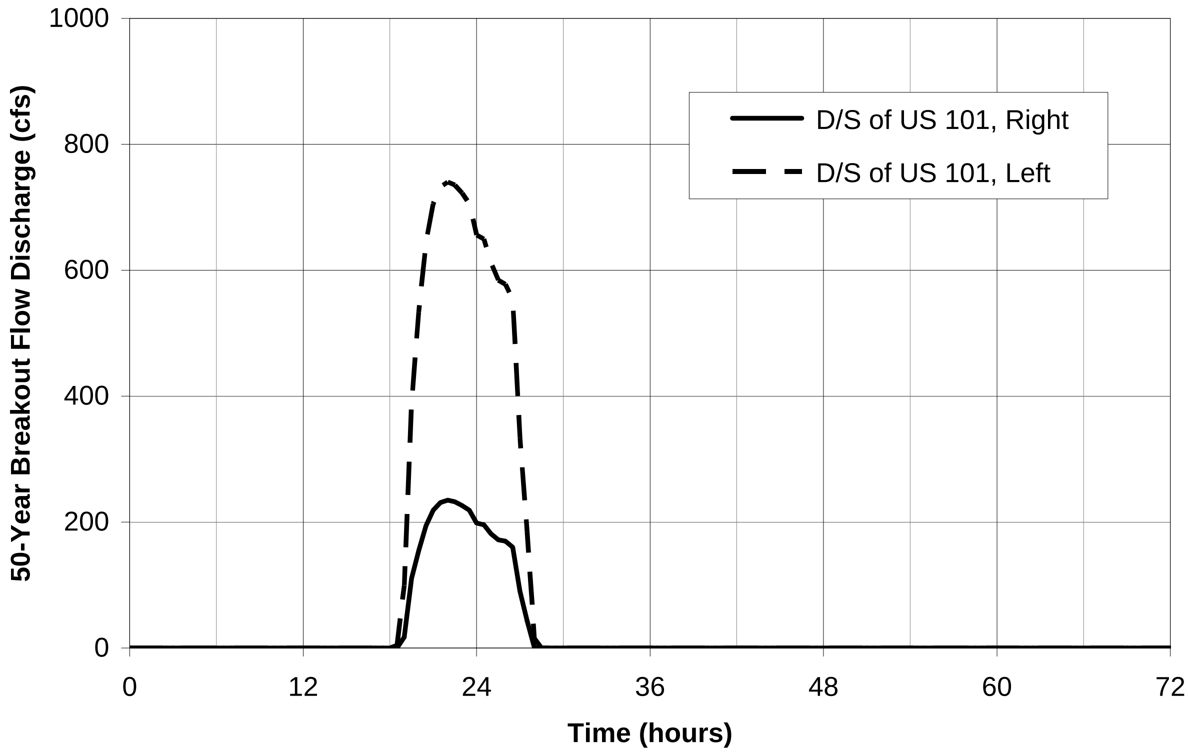
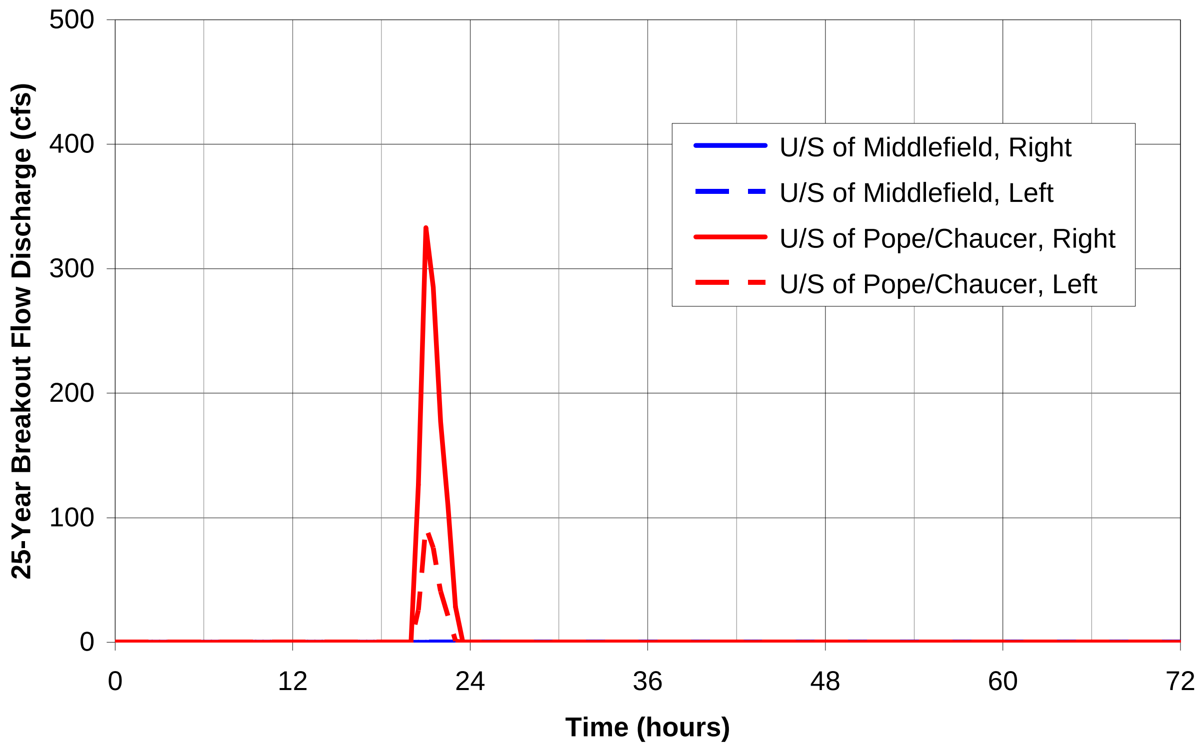


Figure 3-11. Breakout Flow Hydrographs for the 50-Year Flood Event



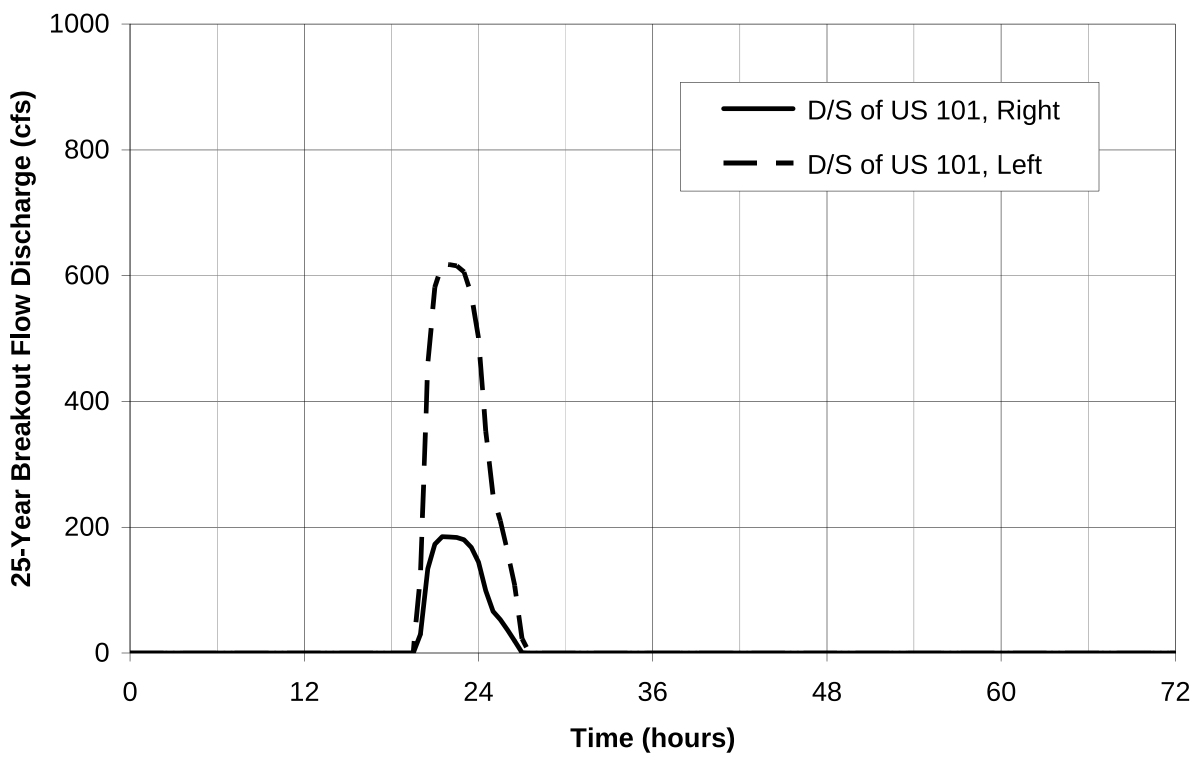


Figure 3-12. Breakout Flow Hydrographs for the 25-Year Flood Event

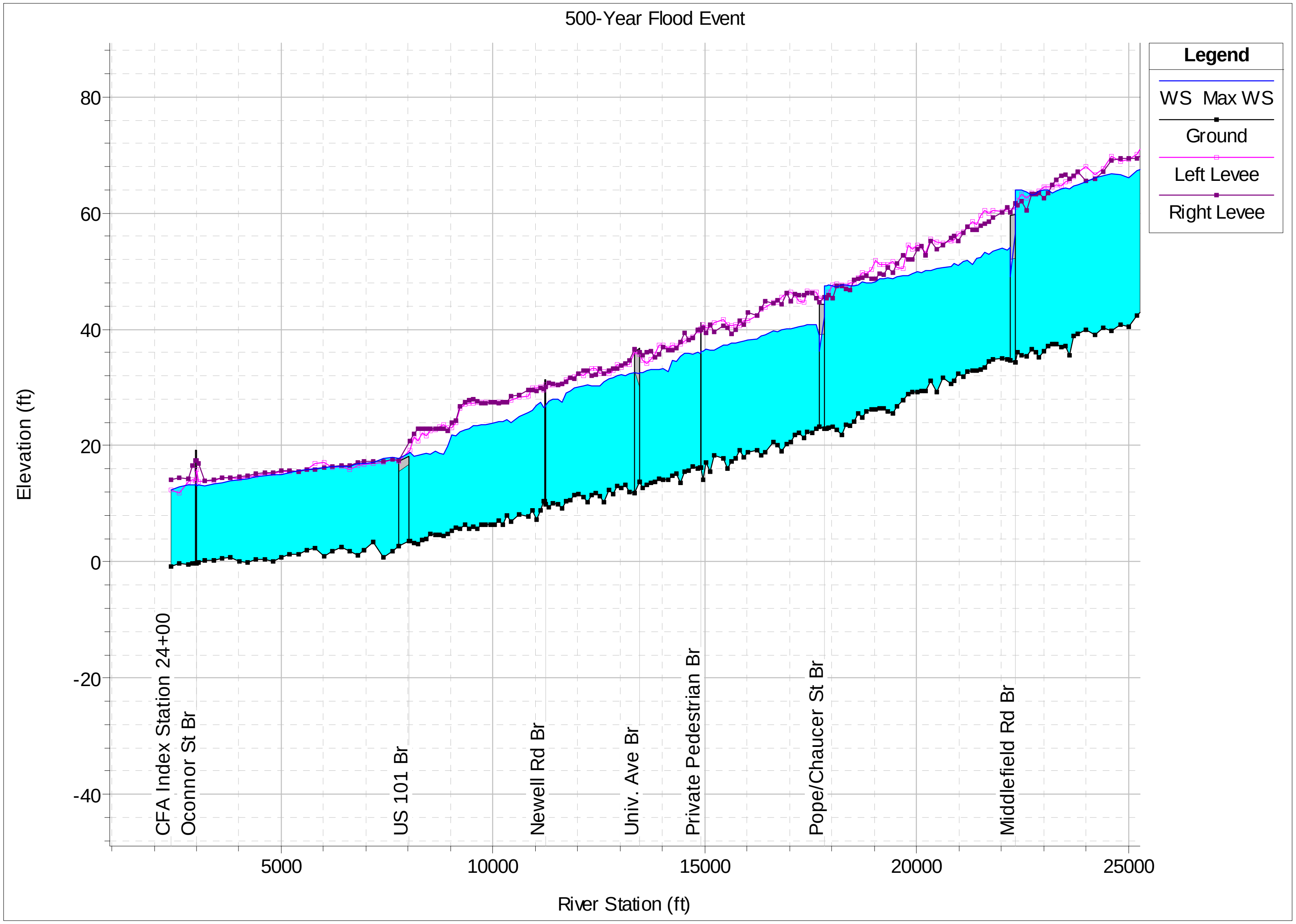


Figure 3-13. 500-Year Water Surface Profile at Flow Breakout Locations

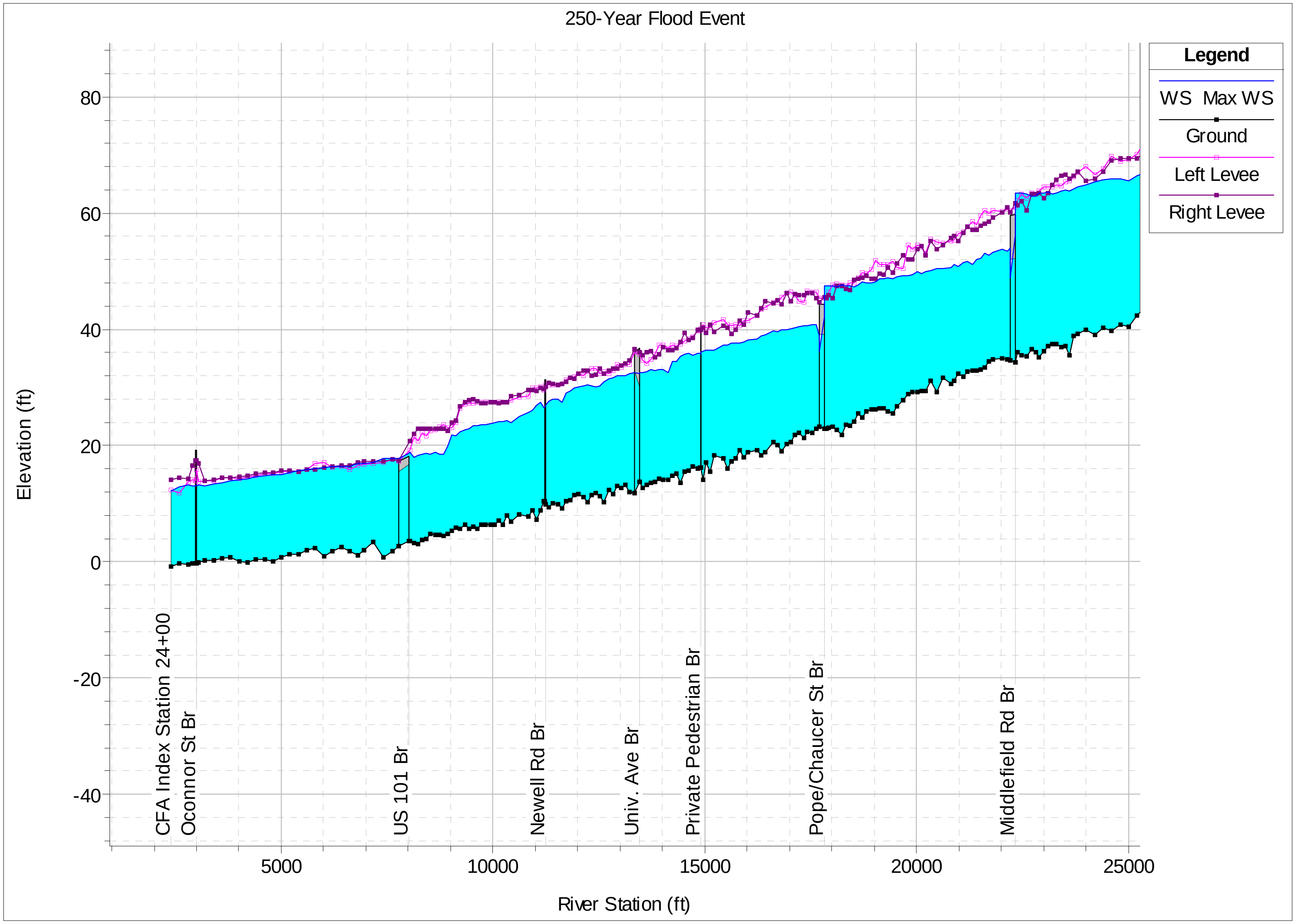


Figure 3-14. 250-Year Water Surface Profile at Flow Breakout Locations

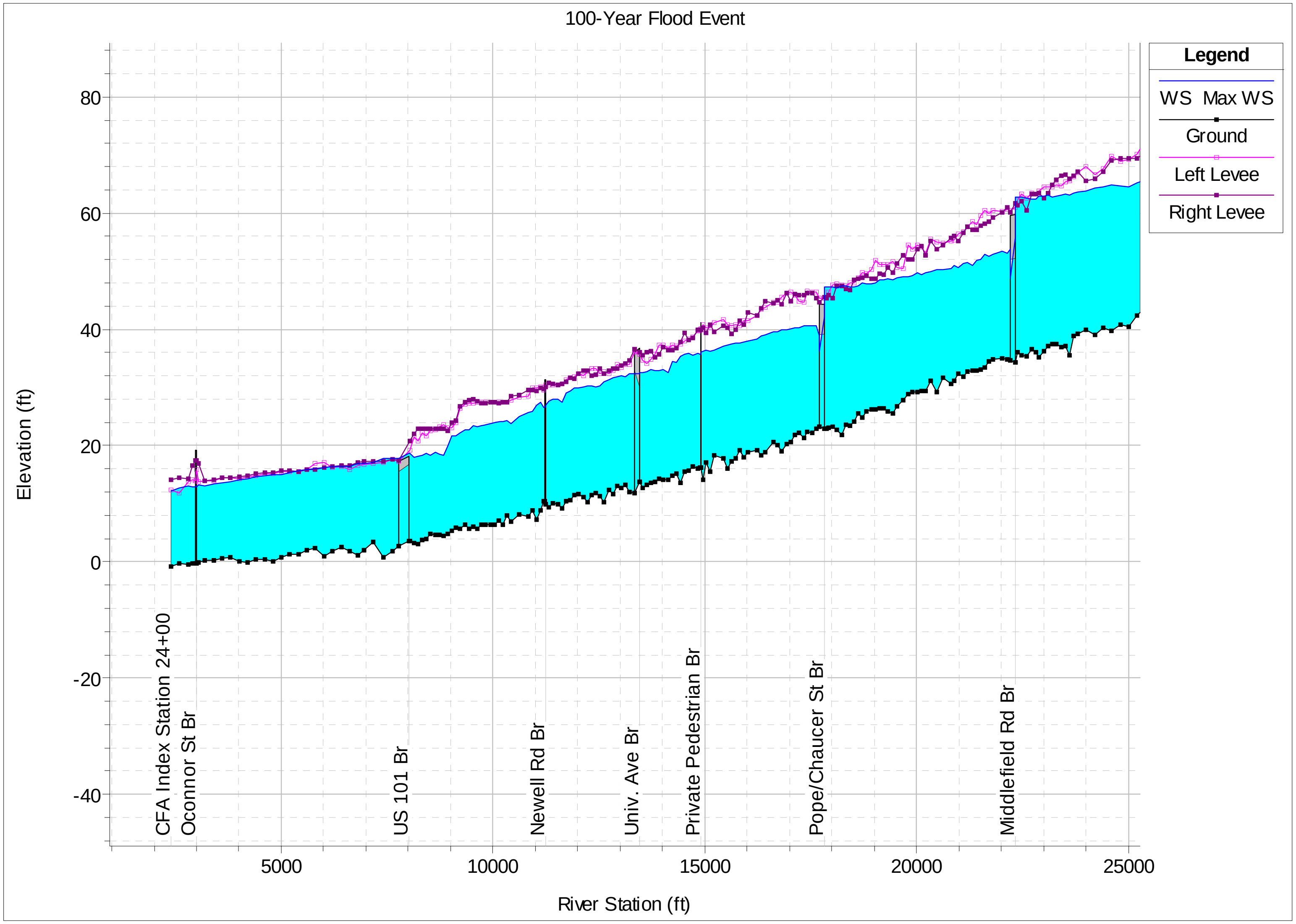


Figure 3-15. 100-Year Water Surface Profile at Flow Breakout Locations

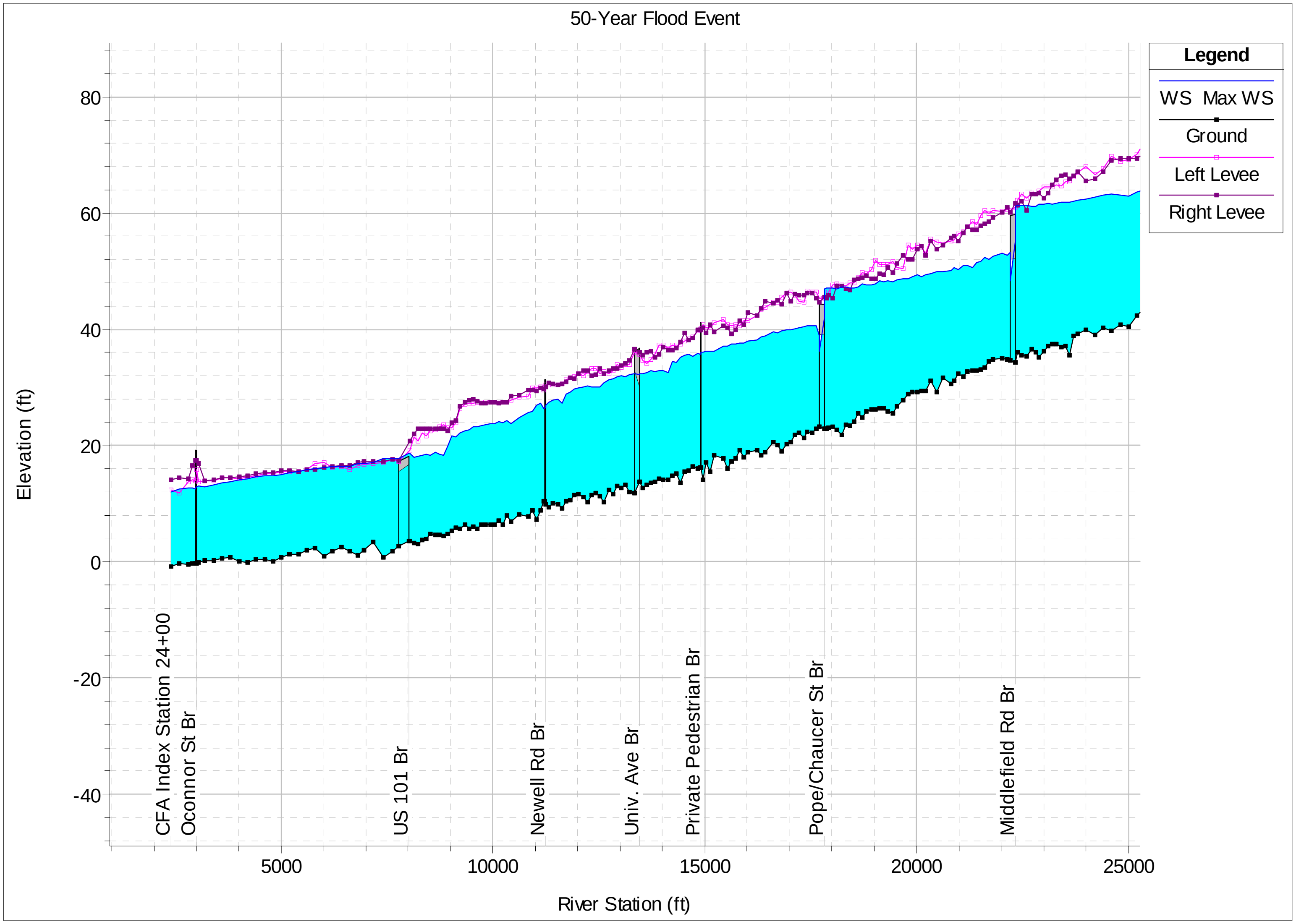


Figure 3-16. 50-Year Water Surface Profile at Flow Breakout Locations

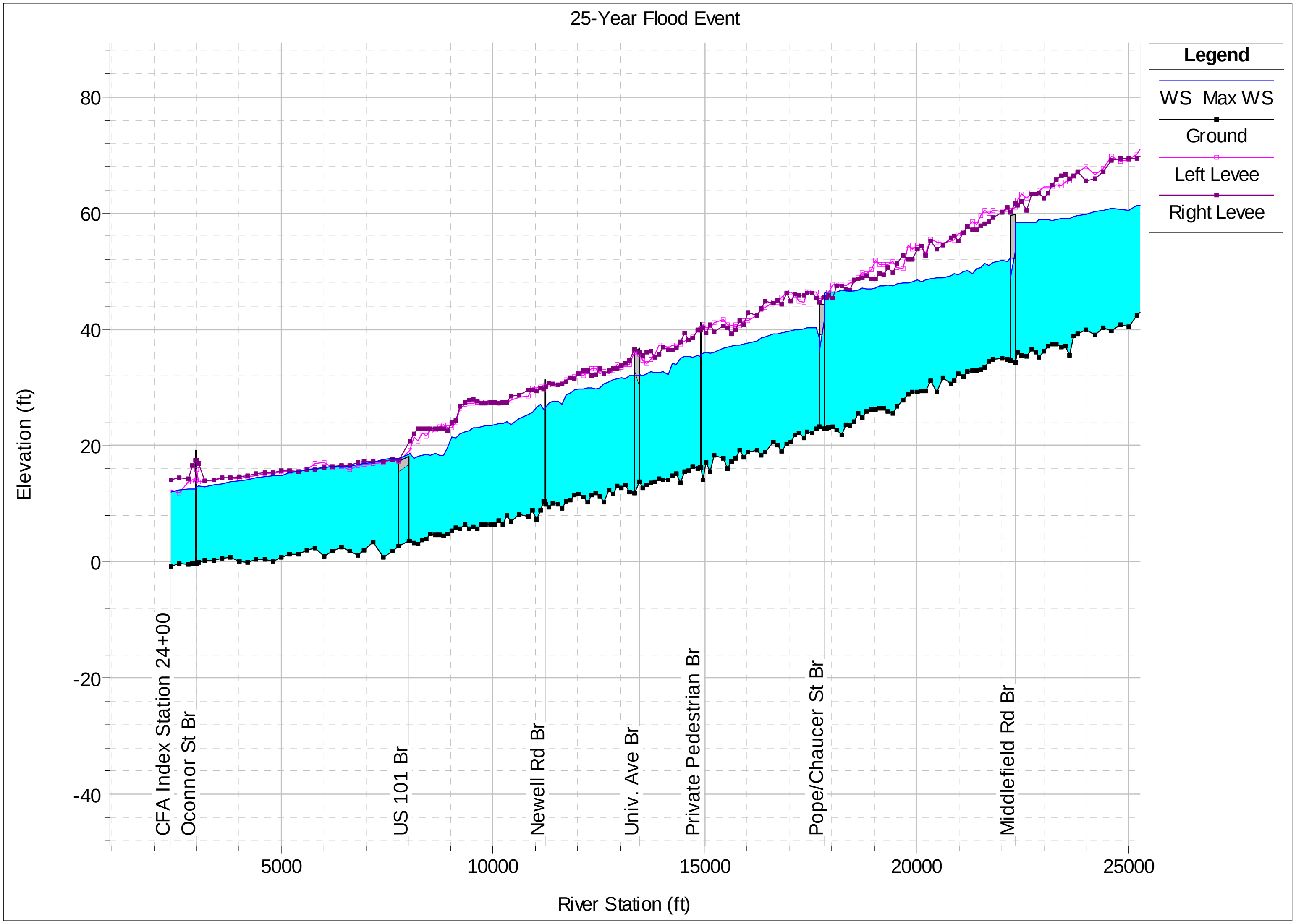


Figure 3-17. 25-Year Water Surface Profile at Flow Breakout Locations

3.4 Computed Water Surface Profiles Upstream of Junipero Serra Blvd

As shown in Figure 3-18, the reach of the San Francisquito Creek that is downstream of Alpine Road and upstream of Junipero Serra Boulevard is confined by residential areas on the left top ground, and by lands/golf course on the right top ground. The model results indicated that the water levels would be higher than the top ground elevations at some locations in this reach, resulting in flooding of the residential areas and lands during the extreme flood events. However, the water flooding these areas will not leave the creek system as these areas are backed by the road embankment and/or hill slopes, as shown in Figure 3-18.

The approach for floodplain delineation specified in the original Scope of Services (USACE, 2009b) requires where flow leaves the bank of the channel should be modeled as a lateral weir (with water leaving out of the creek system). This approach is reasonable for other flow breakout locations along the creek, where water leaving the main channel will flow into the floodplain areas and will not flow back to the channel in a certain period if time. However, this approach is not appropriate for the reach between Alpine Road and Junipero Serra Boulevard, where water flooding the floodplain areas will not be really separated from the main channel.

The correct method of floodplain delineation for this reach shall consist of the following work: (1) extending the cross sections in the existing RAS model to cover the full floodplain areas (residential areas and lands) until reach the road embankment and/or the high ground of the hill slopes, (2) re-running the RAS model with the extended cross sections and with the floodplain areas being modeled as ineffective flow areas or areas with high roughness coefficients, and (3) conducting floodplain delineation based on the computed water surface elevations. Due to the project schedule and budget limits, a simplified method was, however, adopted for the floodplain delineation for this reach after discussing with the Corps. This method will run the existing HEC-RAS model with the cross sections covering the main channel and portion of the floodplain areas and without flow breakout being allowed from the channel even if the computed water levels are higher than the top ground elevations. The computed water levels are then directly used to delineate the floodplain for this reach, rather than using the FLO-2D model with the breakout flow hydrographs as done for the other floodplain areas. The water surface profiles computed for the 100-, 250-, and 500-year flood events are shown in Figure 3-19 through Figure 3-21, respectively. The water levels will exceed the top ground elevations at some locations in this reach during these three flood events. The floodplain delineation and mapping for this reach is presented in a separate report (NHC, 2010).

It is noted that this simplistic approach used for the reach between Alpine Road and Junipero Serra Blvd will impact the accuracy of the floodplain delineation for both this reach and the downstream areas. Because the modeled cross sections did not cover the full extent of the floodplain areas, the flow capacity and the water storage capacity of these floodplain areas and the attenuation of the flood flow due to the floodplain storage were not fully included in the simplistic approach. As a result, the water levels for this reach and the peak flow discharges for the downstream reach might be over estimated. This would result in a more conservative floodplain mapping being estimated for this reach, and for the downstream areas, particularly for the areas upstream of the Middlefield Road. It's also worthy to point out that the impact of the simplistic approach on the accuracy of the floodplain delineation should not be significant considering the limited flow capacity and water storage capacity of these floodplain areas within this reach.

Figure 3-18. Aerial Photo of San Francisquito Creek Upstream of Junipero Serra Blvd

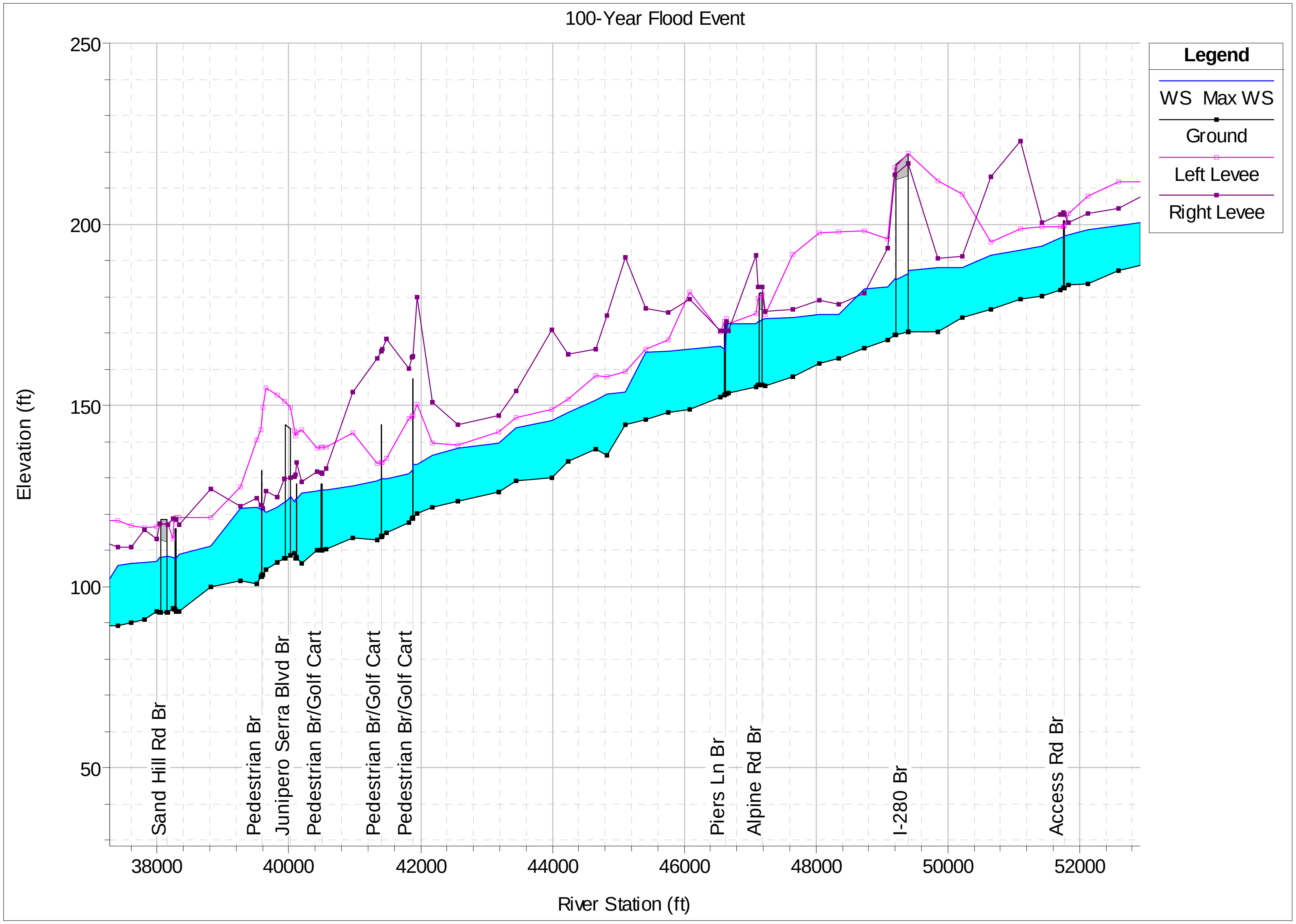


Figure 3-19. 100-Year Water Surface Profile Upstream of Junipero Serra Blvd

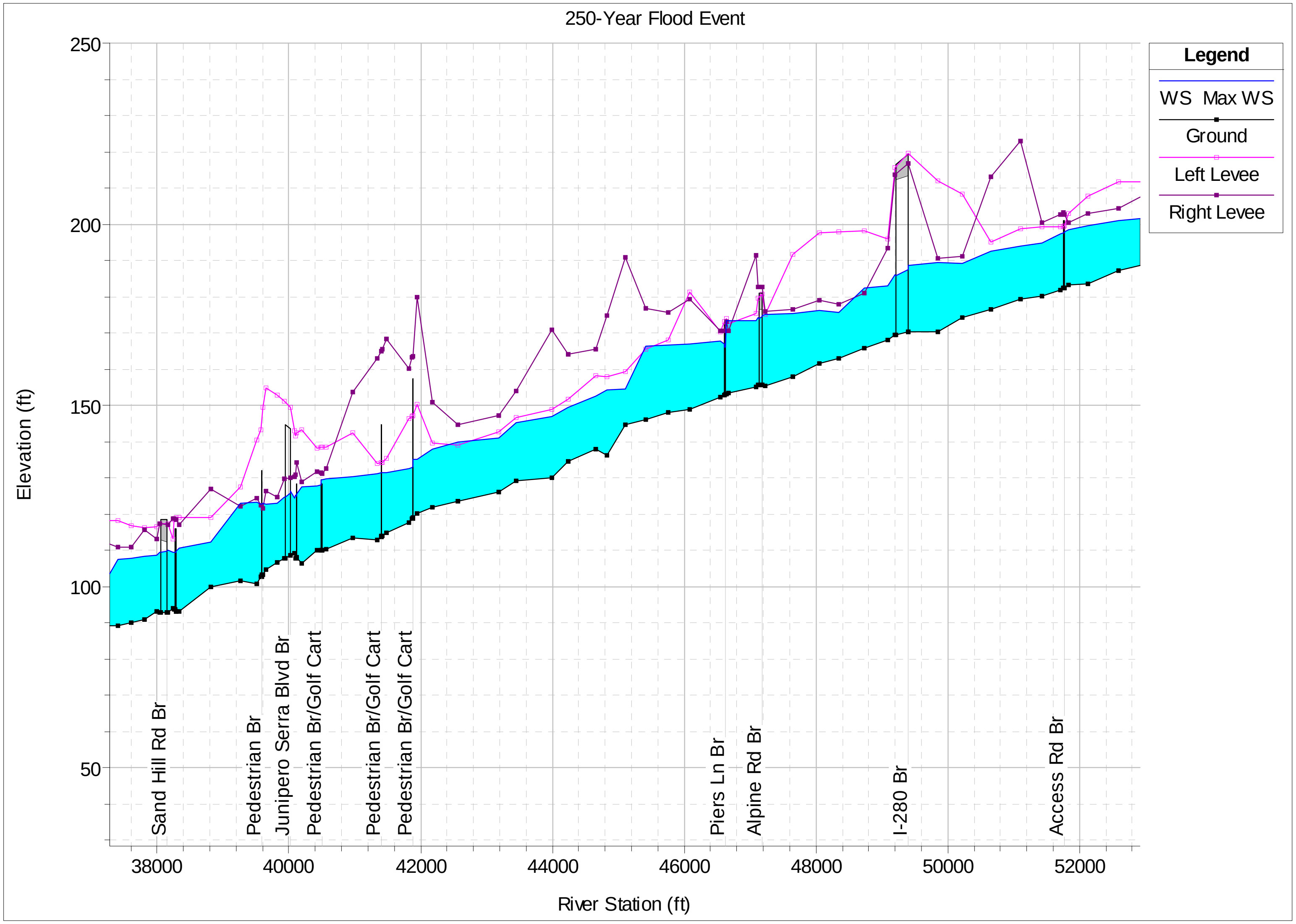


Figure 3-20. 250-Year Water Surface Profile Upstream of Junipero Serra Blvd

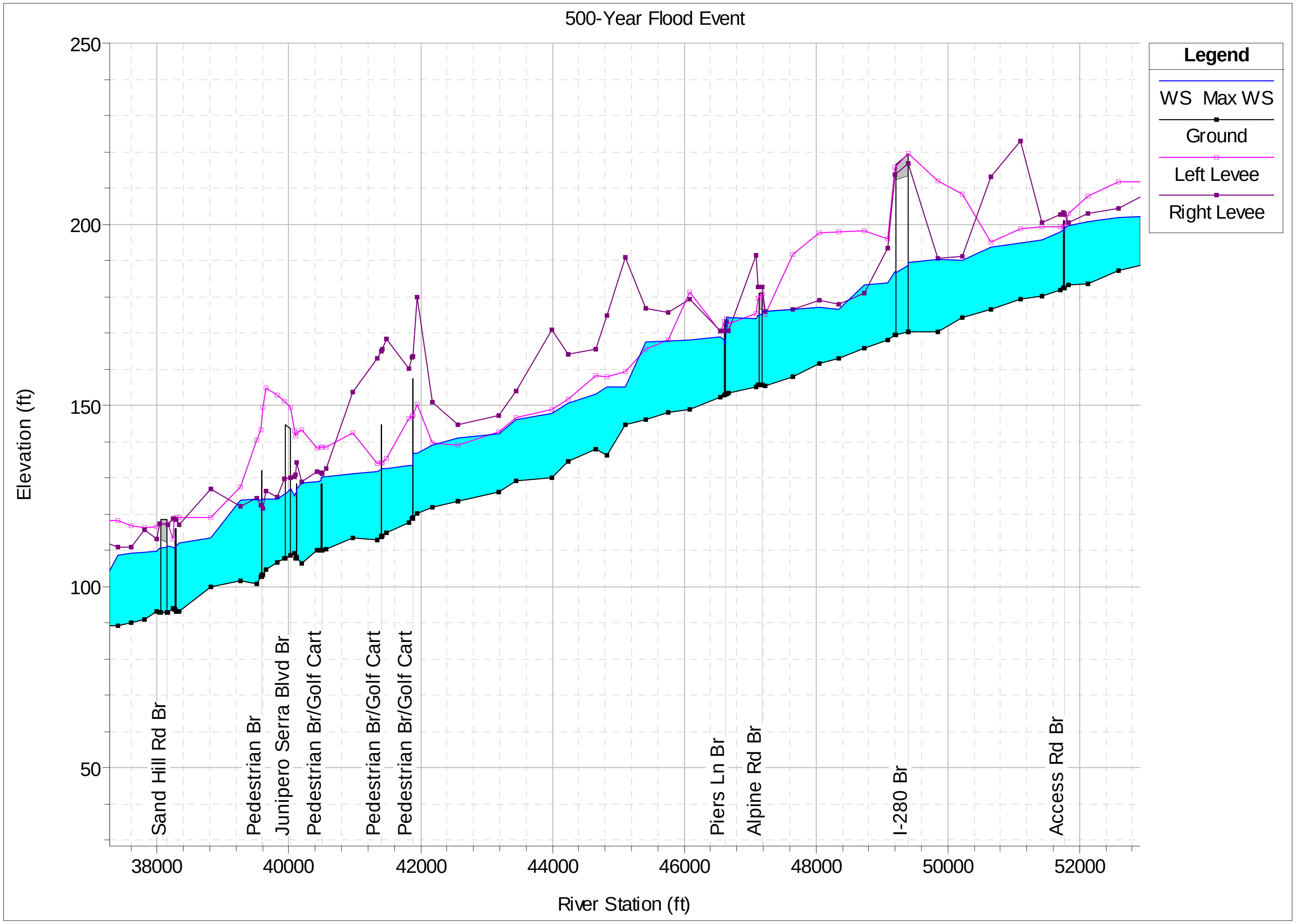


Figure 3-21. 500-Year Water Surface Profile Upstream of Junipero Serra Blvd

4 REFERENCES

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